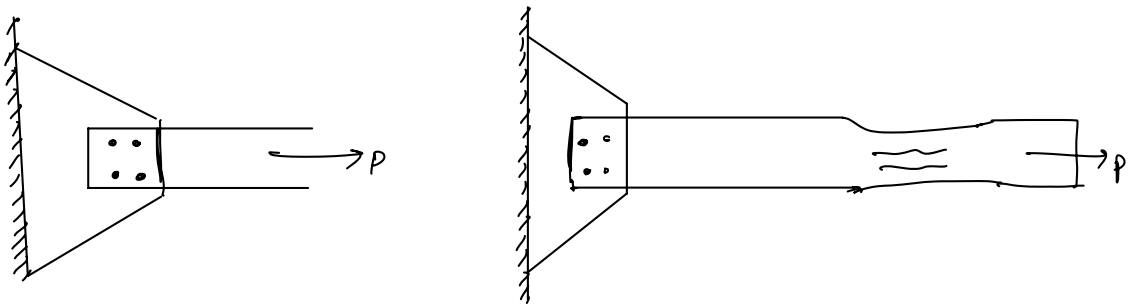
Introduction

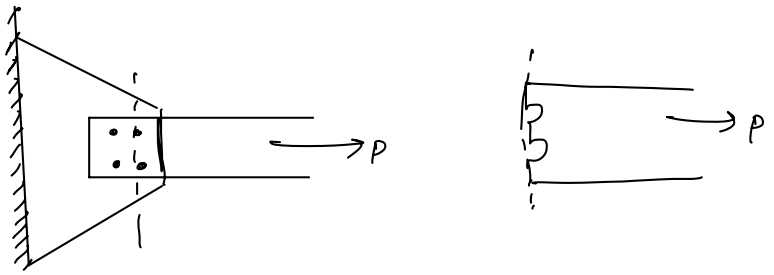
If tensile force is passing through CG then member is designed as a perfectly tension member.

Types of failure

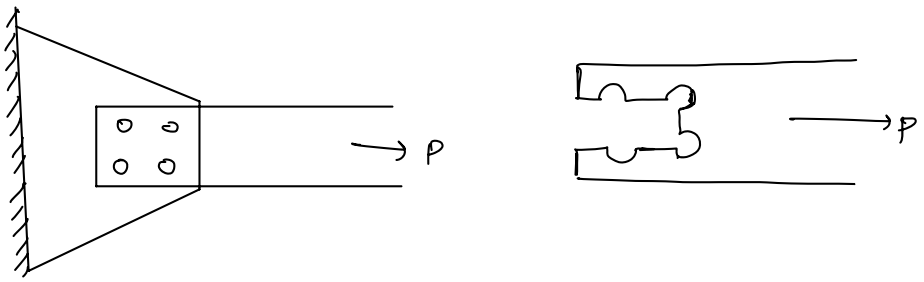
* GROSS SECTION YIELDING



* Rupture of net section



* Block shear failure



* If failure happens in tension member then only tension member failure.

* Gross section Yielding

$$T_{dg} = \frac{f_y}{\gamma_{m0}} \cdot A_g$$

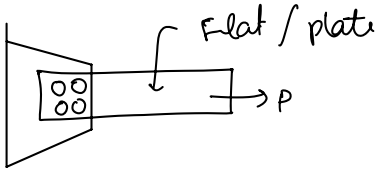
$\gamma_{m0} = 1.1$

A_g = gross Area

* Net section Rupture

- For plates and flats

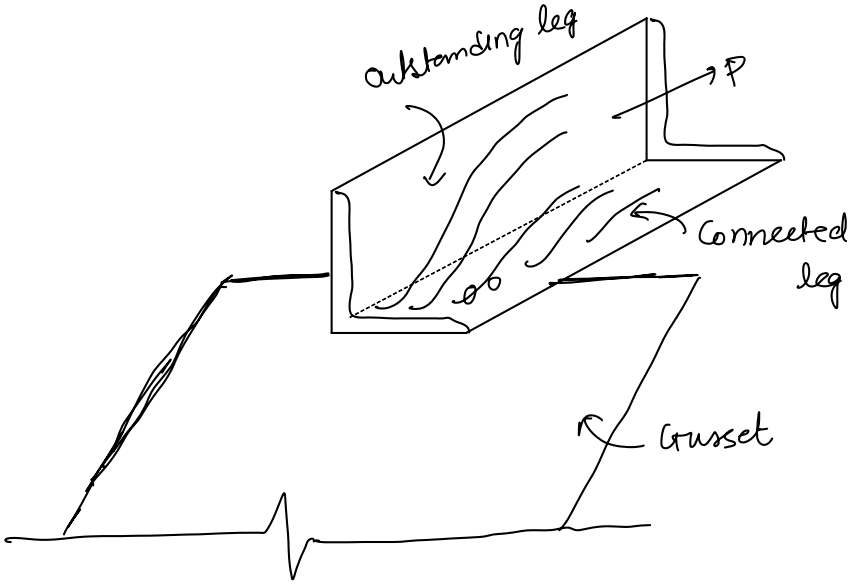
$$T_{dn} = \frac{0.9 f_u}{\gamma_{m1}} \cdot A_n$$

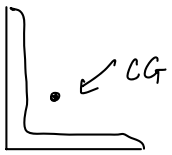


$\gamma_{m1} = 1.25$

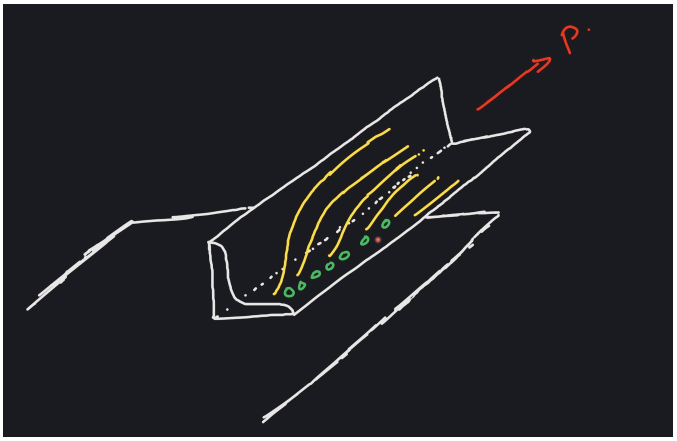
A_n = Net section

- Angle section





* When time the force comes totally on connected leg and very minimal on outstanding leg.



* In case of angle section we have connected leg and outstanding leg. forces transferred from gusset to the connected leg and this force gets transferred as tensile stress over entire section. At the joint, if length of connection is small then connected leg will have higher stress than the outstanding leg The stress in out standing leg lags behind the stress in connected legs. This phenomenon is called shear lag. Due to shear lag effectiveness of outstanding leg is less & hence in case of angles it is better to adopt connection through longer leg in case of unequal legs.

$$T_{dn} = \frac{0.9 f_u}{\gamma_{m_1}} \cdot A_{nc} + \frac{\beta f_y}{\gamma_{m_0}} \cdot A_{go}$$

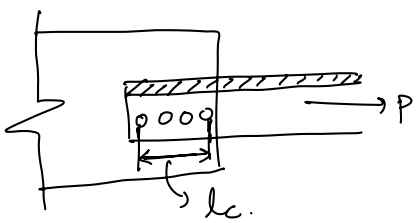
A_{nc} → Area of connected leg (Net)

A_{go} → Area of outstanding leg (Gross)

where,

$$\beta = 1.4 - 0.076 \frac{w}{t} \cdot \frac{f_y}{f_u} \cdot \frac{b_s}{l_c}$$

l_c → length of connection measured from first bolt to last bolt along the direction of force.

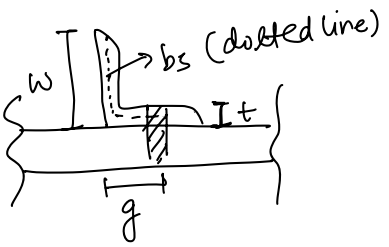


$$0.7 \leq \beta \leq \frac{f_u/\gamma_{m_1}}{f_y/\gamma_{m_0}}$$

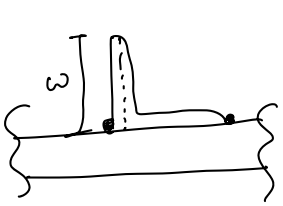
w = length of outstanding leg

t = thickness of angle section

b_s = shear leg width and it is measured from the farthest point on the outstanding leg to the nearest line of bolt or weld in the connected leg. along the centroidal



$\{ bs = w + g - t \}$.



$\{ bs = w \}$

* In welded both sides must be welded.

* Note :- Increasing length of connection will reduce shear lag effect.

* <u>Block shear failure</u>

<u>Slenderness Ratio</u>

Slenderness Ratio $= \lambda = \frac{l_{eff}}{\gamma}$, where, $\gamma = \sqrt{\frac{I}{A}}$

There is no buckling or stability problem in case of tension member, however reversal of stress may be possible so slenderness ratio is restricted.

Description	Maximum Slenderness Ratio
1. A tension member in which reversal of direct stress occur due to loads other than winds & seismic forces	180

(For example, member of truss on railway bridge)

2. A member normally acting as a tie (tension) in a roof truss or a bracing system not considered effective when subjected to possible reversal of stress into comp. resulting from action of wind or earthquake

350

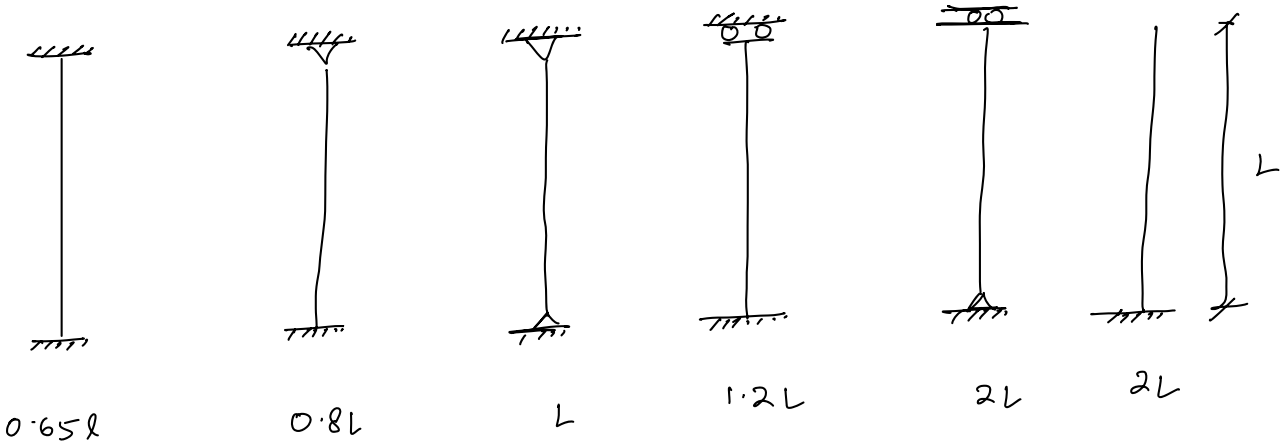
3. Member always under tension (other than pretensioned member)

400

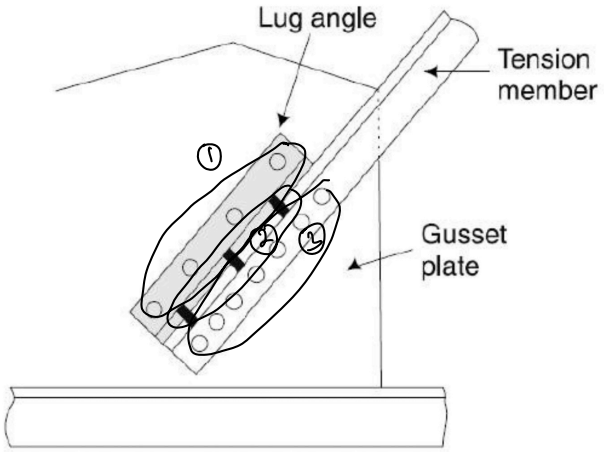
{sagging due to self weight}

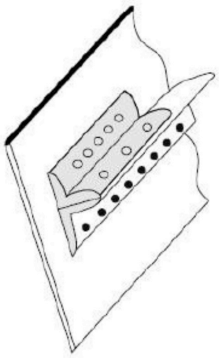
{Table 3 of IS 800 : 2007, page -20}

* Effective length



* Lug Angle





* Both legs got connected thus no shear lag effect

Bolt ③ - Main member to Gusset plate

Bolt ② - Outstanding leg of main member to outstanding leg of Lug Angle

Bolt ① - Lug angle to gusset plate

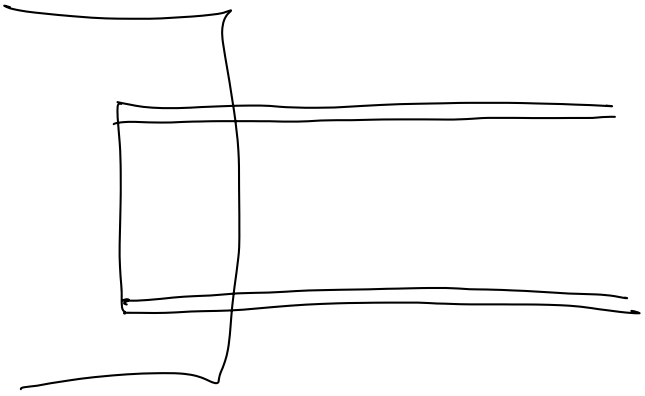
1. Lug Angle is provided to reduce the length of connection & also negates the shear lag effect.

2. For connection to the efficient, bolts connecting the outstanding leg of main member with lug angle should start in advance to all other bolts. This will ensure that force gets efficiently transferred from outstanding leg to lug angle.

3. The main member will be assumed to have no shear lag effect.

4. Tensile strength of main member along the min net section is calculated

5. Minimum number of bolts connecting the lug angle with gusset plate should be 2

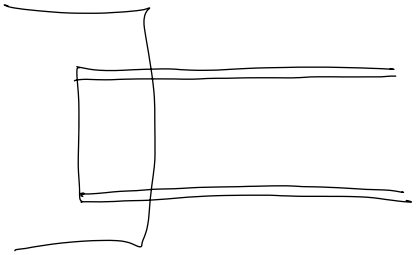


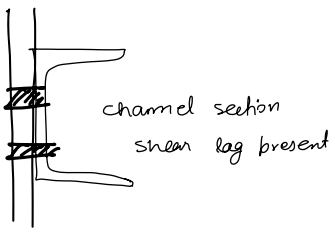
channel section
shear lag present

If main member is angle section

- Lug Angle and its connection width gusset plate should be designed for force $\geq$ 1.2 times force in outstanding leg
- connection of outstanding leg of main member with lug angle should be designed for force $\geq$ 1.4 times force in outstanding leg of main member

If main member is channel section:-





channel section
shear lag present

- $>$1.1 times
- $\geq$ 1.2 times

} points are same.

Tension Member lecture

Design of tension members (Angle section)

* Calculate factored load
* Calculate approx net sectional area

$$T_u = \left(\frac{0.9 f_u}{\sqrt{3}\,\gamma_{m1}} \cdot A_{net} \right) \alpha$$

$$A_{net} = \frac{T_u}{\alpha \cdot \left(\frac{f_u}{\sqrt{3}\gamma_{m_1}}\right)}$$

$\alpha = 0.6$ (for 1 or 2 bolts) $= 0.8$ (for welded)

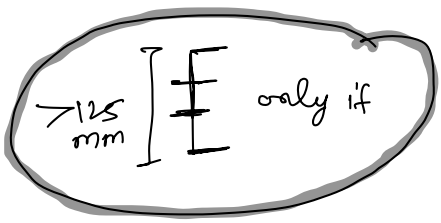
$= 0.7$ (for 3 bolts)

$= 0.8$ ($\geq$ 4 bolts)

* Increase Anet by 20 to 30% to obtain gross area
* Provide suitable connection to support
* Calculate actual capacity of section in yielding rupture & block shear
* Capacity of step 5 should be higher than load of step 1. Otherwise provide larger section and repeat steps 4 and 5.

Note :-

1. 2 lines of bolts can be provided on 1 leg of angles only when connected leg > 125 mm
2. 2 lines of bolts can be provided on the web of channel section only if depth ≥ 150 mm
3. In case of angle section prefer to connect longer leg.



Question

Design a diagonal member of truss subjected to $T_u = 300KN$. length of diagonal = 3m, The tension member is connected to gusset plate of 16mm Grade of bolt is 4.6, Steel section is Fe410

Sectional property of ISA $\boxed{125 \times 95 \times 8}$

$A = 17cm^2$

$C_{xx} = 3.8cm$

$C_{yy} = 2.32cm$

$\left\{ r_{uu} = 4.41 \text{ cm}, \; r_{vv} = 2.05 \text{ cm} \right\}$

$r_{xx} = 3.97$ cm

$r_{yy} = 2.81$ cm.

Solution

Factored load = 300 KN = T_u

Assuming $\alpha = 0.8$ for number of bolts > 4.

$$A_{net} = \frac{T_u}{\frac{\alpha \cdot F_u}{\gamma_{m1}}} = \frac{300 \times 10^3}{\frac{0.8 \times 410}{1.25}} = 1143.29 mm^2$$

Increasing by 20%,

$$A_{gross} = 1.2 \times A_{net} = 1270.3 \times 1.2 = 1524.39 mm^2$$

Select suitable section from SP6.

Providing ISA 125X95X8

* Check slenderness

$$\lambda = \frac{l_{eff}}{r_{min}} = \frac{3 \times 10^3}{2.05 \times 10} = 146.34 < 180$$

Thus OK.

* Design of connections

Assuming dia of bolt = 20mm (experience)

$$\text{capacity in shear} = \frac{f_u}{\sqrt{3}\,\gamma_{mb}} \times A_{net} = \frac{400}{\sqrt{3} \times 1.25} \times 0.78 \times \frac{\pi}{4} \times 20^2$$

$$= 45.27 KN$$

* Capacity in bearing

$$= \frac{2.5\, K_b\, dt\, f_u}{\gamma_{mb}}$$

K_b = minimum $\begin{cases} \frac{e}{3d_0} = \frac{1.5d_0}{3d_0} = 0.5 \\ \frac{p}{3d_0} - 0.25 = \frac{2.5d}{3d_0} - 0.25 \approx 0.5833 \\ \frac{f_{ub}}{f_u} = 0.975 \\ 1 \end{cases}$

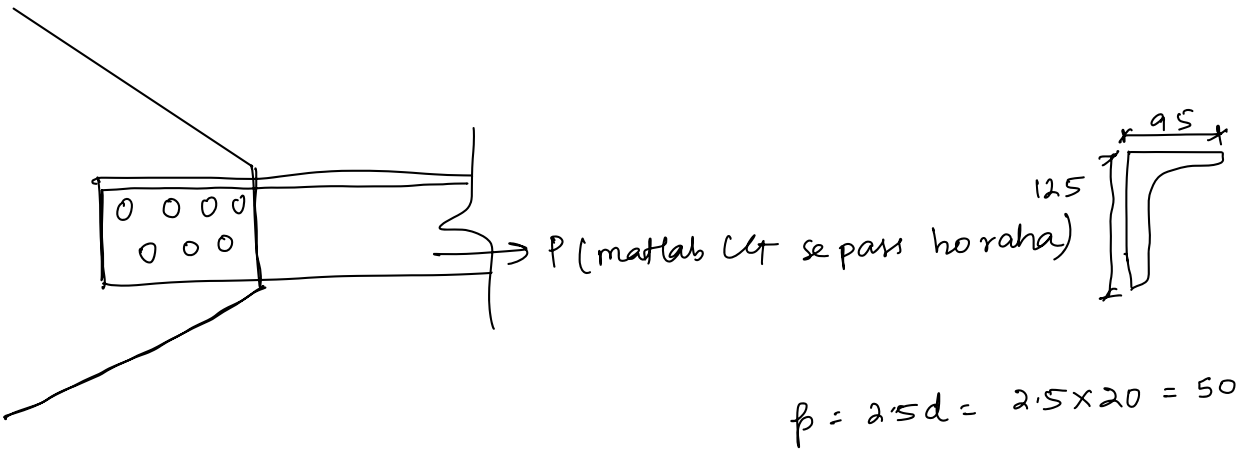
$= 0.5$

$$= \frac{2.5 \times 0.5 \times 20 \times 8 \times 410}{1.25} = 65.6\ KN$$

Capacity of bolt = 45.87KN

So number of bolts required = $\frac{300}{45.27} = 6.62$

Provide 7 bolts in staggered arrangement



$p = 2.5d = 2.5 \times 20 = 50mm$

$e = 1.5d_0 = 1.5(22) = 33 \approx 35mm$

Staggered bolting is generally better than chain bolting.

Now, tension capacity of member

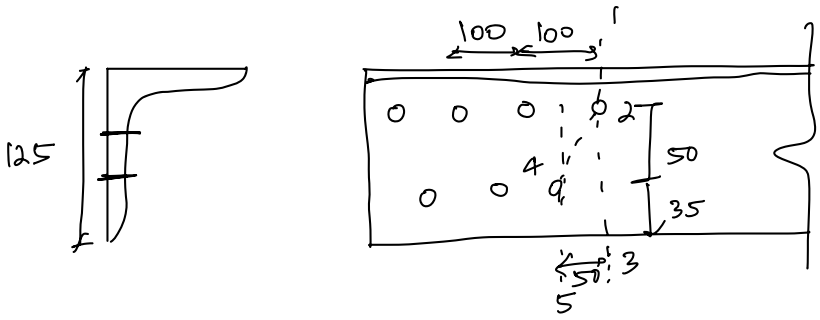
(*) Gross yielding

$$T_{dg} = \frac{f_y}{\gamma_{m0}} A_g = \frac{250}{1.1} \times 1700 = 386.36\ KN \quad > 300KN$$

(*) Net section rupture

$$T_{dn} = \frac{0.9 f_u}{\gamma_{m1}} \times A_{nc} + \frac{\beta f_y}{\gamma_{m0}} A_{go}$$

For A_{nc} & A_{go}



$$A_{nc}(1\text{-}2\text{-}3) = \left(125 - \frac{8}{2} - 22\right) 8 = 792\ mm^2$$

$$A_{nc}(1\text{-}2\text{-}4\text{-}5) = \left(125 - \frac{8}{2} - 2\times 22 + \frac{50^2}{4\times 50}\right) \times 8 = 716\ mm^2$$

↓ Formula from code book page 33.

$$A_{go} = \left(95 - \frac{8}{2}\right) \times 8 = 728\ mm^2$$

$w = 95$

$b_s = 95 + 40 - 8 = 127$

$L_c = 300\ mm$ (distance between 1st & last bolt)

$$\beta = 1.4 - 0.076 \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{95}{8} \times \frac{250}{410} \times \frac{127}{300} = 1.167$$

$$0.7 \leq \beta \leq \left(\frac{410/1.25}{250/1.1} = 1.44 \right)$$

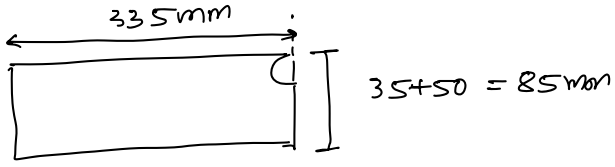
Thus, $\beta = 1.167$

$$T_{dn} = \frac{0.9 \times 410}{1.25} \times 716 + \frac{1.167 \times 250 \times 728}{1.1}$$

$$= 404.448 \text{ kN} > 300 \text{ kN} \quad (\text{Thus OK})$$

* Capacity of block shear

- Shear yielding + tension fracture



$$T_{b1} = \frac{f_y}{\sqrt{3}\gamma_{m0}} \times A_g + \frac{0.9 f_u}{\gamma_{m1}} \times A_{tn}$$

$$A_{vg} = 335 \times 8 = 2680 \text{ mm}^2$$

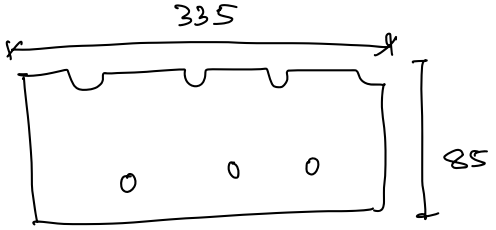
$$A_{tn} = \left(85 - \frac{22}{2}\right) \times 8 = 592 \text{ mm}^2$$

$$= \frac{250 \times 2680}{\sqrt{3} \times 1.1} + \frac{0.9 \times 410 \times 592}{1.25}$$

$$= 526.417 \text{ kN}$$

[yielding first hota hai]

- Shear fracture + tension yielding



$$A_{vn} = (335 - 3.5 \times 22) \times 8$$

$$= 2064 \text{ mm}^2$$

$$A_{tg} = 85 \times 8 = 680 \text{ mm}^2$$

$$T_{db2} = \frac{0.9 f_u}{\sqrt{3}\gamma_{m1}} A_{vn} + \frac{f_y}{\gamma_{m0}} A_{tg}$$

$$= \frac{0.9 \times 410}{\sqrt{3} \times 1.25} \times 2064 + \frac{250}{1.1} \times 680 = 506.32 \text{ kN}$$

Thus member is safe in tension.

Question 2

A tension member subjected to serviceable load of 300 KN. Design the section and its connection with 16mm thick gusset plate. The length of connection is limited to 340mm. E250, Bolt 4.6.

Factored force $= 1.5 \times 300 = 450$ KN

$$A_{net} = \frac{T_u}{\frac{\alpha \times f_u}{\gamma_{m1}}} = \frac{450 \times 10^3}{\frac{0.8 \times 410}{1.25}} = 1714.94 \text{ mm}^2$$

Assuming number of bolts ≥ 4

Increasing by 20% = $A_g = 1.2 \times A_{net} = 1.2 \times 1905.48$

$= 2286.58 \text{ mm}^2$

Providing ISA $150 \times 75 \times 12$ (from SP6)

$$\text{Provided } A_g = \left(150 - \frac{12}{2}\right) 12 + \left(75 - \frac{12}{2}\right) 12 = 2556 \text{ mm}^2$$

Capacity of bolt in shear $= \frac{f_{ub} \times A_{nb}}{\sqrt{3}\,\gamma_{mb}} = 45.27$ KN (assuming 20mm bolts)

Capacity in bearing $= \frac{2.5 K_b d t f_u}{\gamma_{mb}}$

$K_b = 0.5$ (as in previous question)

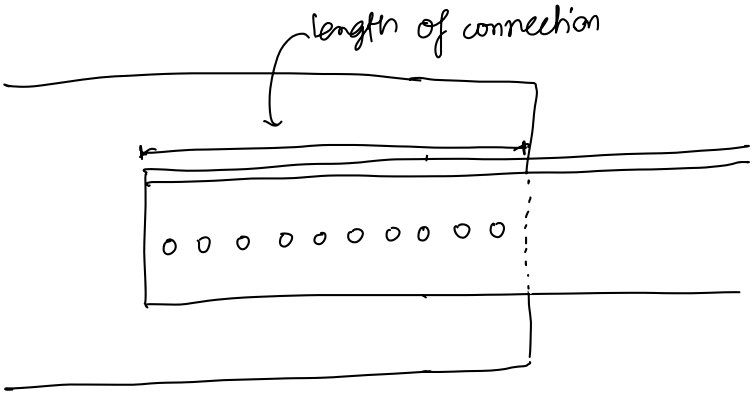
$$= \frac{2.5 \times 0.5 \times 20 \times 12 \times 410}{1.25} = 98.4 \text{ KN}$$

BOLT VALVE = 45.27 KN

No of bolts required $= \frac{T_u}{45.27} = \frac{9.94}{} \approx 10$ bolts

pitch = 2.5d = 50mm

edge / distance = 1.5do = 1.5 x 22 = 33mm ≈ 35mm.



length of gusset plate required

= 9x pitch + 2x end distance

= 9x50 + 2x35

= 520mm > 340mm (limited)

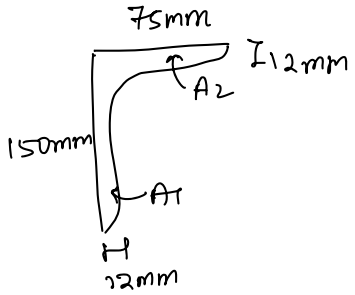
So, providing lug angle, (TOTAL 10 Bolts)

Providing 6 bolts in main member

length of gusset plate required = 5x50+70

= 320mm < 340mm

Design of lug Angle



$A_1 = \left(150 - \frac{12}{2}\right)12 = 1728mm^2$

$A_2 = \left(75 - \frac{12}{2}\right)12 = 828mm^2$

Force in outstanding leg = $\left(\frac{A_2}{A_1+A_2}\right) T_u = \frac{828}{1728+828} \times 450 = 145.77 KN$

Lug angle should be designed for = 1.2x145.77 = 174.92 KN

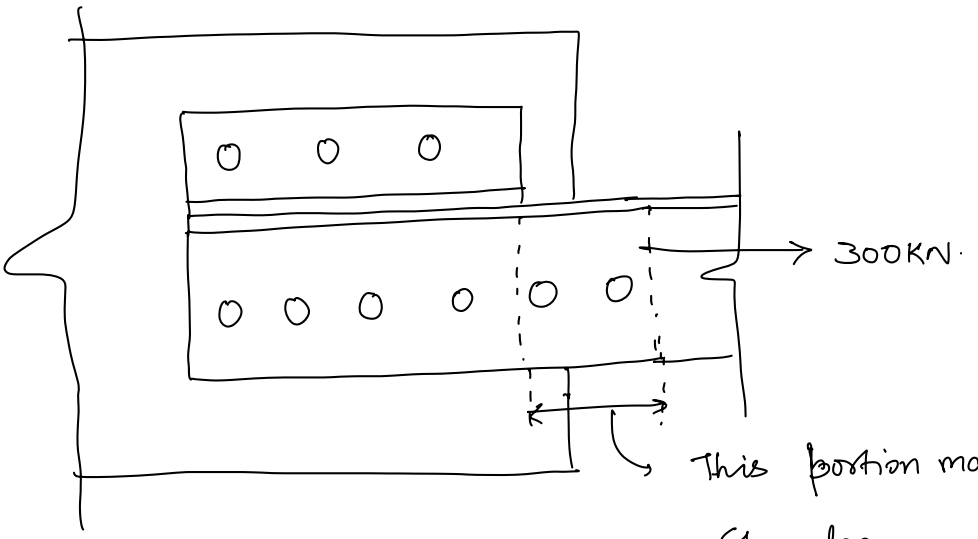
No of bolts required = $\frac{174.92}{45.27} \approx 3.86 = 4$ bolts

Anet for Lug Angle = $\frac{T_u \text{ of Lug Angle}}{\alpha \times \frac{0.9 f_{ub}}{\gamma_{mb}}} = 759.2 mm^2$

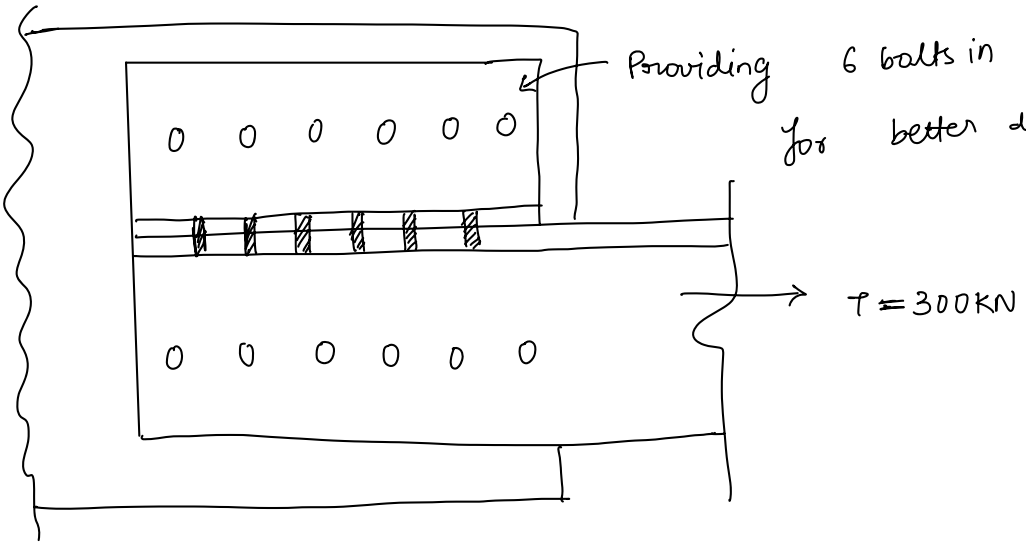
$A_g = 1.2 \times 759.2\ mm^2 = 888.82\ mm^2$

Providing 75 x 75 x 10 ISA (from SP6)

$A_g = \left(75 - \frac{10}{2}\right) \times 10 \times 2 = 1400\ mm^2$



So, for better detailing, providing connection as follows



Bolts of outstanding leg of main member and lug angle shall be designed for = $1.4 \times 145.77 = 204.07$ KN

Number of bolts required = $\frac{204.078}{45.27} = 4.5 \approx 6$ bolts

Check of capacity of Tension Member:

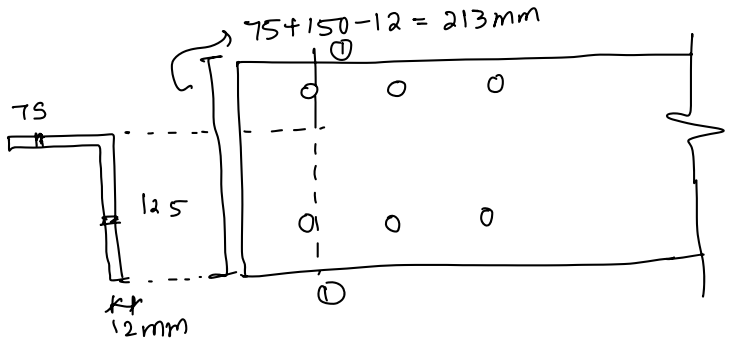
(*) $T_{dg} = \frac{f_y}{\gamma_{m0}} \cdot A_g = \frac{250}{1.1} \times 2556 = 580.9\ kN$

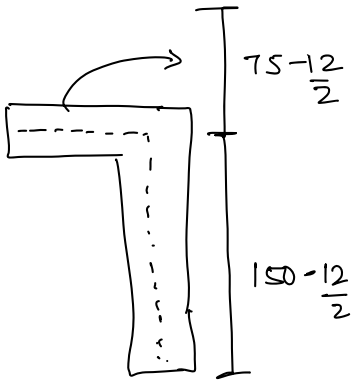
$> 450 kN$

Thus OK

(*) Net section rupture $= \frac{0.9 f_u A_n}{\gamma_{m_1}} = \frac{0.9 \times 410 \times 2028}{1.25} =$

(Leg Angle || No shear lag)

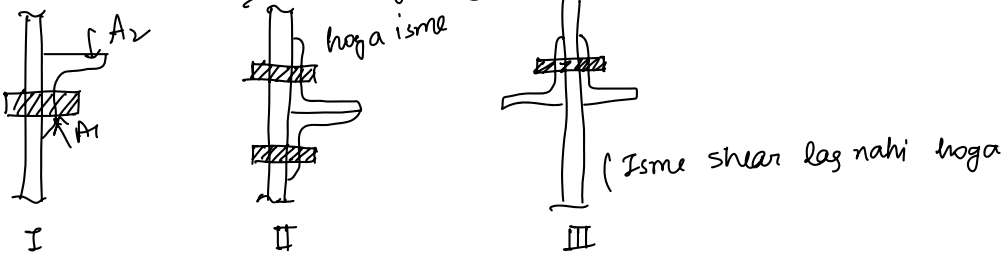




(Total length = 150+75−12)

$A_{net(1-1)} = (213 - 2 \times 22) 12 = 2028 mm^2$

* Net Area of angle section as per IS 800 : 1984



A_1 = Net Area of connected leg

A_2 = Net Area of outstanding leg.

Case I $\quad A_{eq} = A_1 + KA_2 \qquad K = \frac{3A_1}{3A_1 + A_2}$

Case II $\quad A_{eq} = A_1 + KA_2 \qquad K = \frac{5A_1}{5A_1 + A_2}$ } only for tack rivited

Caso III

$A_{CG} = 2(A_1 + A_2)$